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SUBSURFACE INVESTIGATION REPORT

BUILDING 825
STORM WATER DRAINAGE UPGRADE

WHEELER AIR FORCE BASE, OAHU, HAWAII

for

OKAHARA & ASSOCIATES, INC.

by

FEWELL GEOTECHNICAL ENGINEERING, LTD.

SEPTEMBER 30, 2009

FEWELL GEOTECHNICAL ENGINEERING, LTD.

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File 2945.01
September 30, 2009

Okahara & Associates, Inc.
200 Kohala Street
Hilo, Hawaii 96720-4323

Attention: Mr. Masahiro Nishida, Vice President
Ms. Lennie Okano-Kendrick

Subject: **Subsurface Investigation Report**
Building 825 Storm Water Drainage Upgrade
Wheeler Air Force Base, Oahu, Hawaii

We have completed a subsurface investigation and a percolation test for the Building 825 Storm Water Drainage Upgrade project at Wheeler Air Force Base, in Wahiawa, Oahu, Hawaii. This report presents our findings and conclusions. This work was completed in general accordance with our February 29, 2009 Proposal and your May 7, 2009 authorization to proceed.

A number of clearances had to be obtained to gain access into the site to drill the borings and percolation holes, and Fewell Geotechnical Engineering, Ltd. (FGE) was assisted in this endeavor by the Hawaii Air National Guard. However, significant time was necessary to obtain access to the site, which resulted in a delay in the performance of the field work and the subsequent preparation of this report.

Purpose and Scope - The Building 825 Storm Water Drainage Upgrades project will include capping an existing access manhole to a tunnel behind Building 825 and installing an infiltration bed in an open area to the south of Building 825. A subsurface investigation was undertaken to assist Okahara & Associates, Inc. (OAI) with the geotechnical aspects of the design and construction of the cover for the manhole and to perform a percolation test in the area of the infiltration beds for the drainage improvements.

Our work included drilling a test boring adjacent to the access manhole to the tunnel, sampling and testing the soils obtained, evaluating the soil conditions with regard to the proposed manhole cover, and presenting our findings in this report. In addition, a boring was drilled in the vicinity of the infiltration beds to determine the general subsurface conditions in this area, and a percolation test was performed. The percolation test results are presented in this report. The evaluation of the percolation test results and the design of the infiltration system will be performed by others and is not a part of our work.

A Site and Boring Location Plan, together with the logs of the borings and percolation hole, and the laboratory test results are included in the attached Appendix.

Project Considerations – Building 825 is on the southern side of Santos Dumont Road, just east of West Hill at Wheeler Air Force Base in Wahiawa. It is within the restricted area of the adjacent airfield and is secured with 10-foot high chain link fencing and video cameras. Santos Dumont Road is on its northern side and aircraft hardstands are on its southern side. The general area of Building 825 is shown on the Project Location Map, Figure 1, in the Appendix.

The site of Building 825 is about 15 feet below the level of the adjacent Santos Dumont Road, which is aligned in a general east-west direction in this area. Access is via a sloping asphalt driveway which leads from the southern side of Santos Dumont Road to the parking lot for Building 825. The Building is to the east of the parking lot and within about 35 feet of the right-of-way for the road. The security fence for the airfield passes along the edge of the right-of-way for Santos Dumont Road and parallel to the road.

The level of the road in the area adjacent to Building 825 is estimated at between Elev. 843 and Elev. 844, while the area between Santos Dumont Road and Building 825 is at about Elev. 833. A 12-foot high concrete retaining wall supports the grade difference between the road and the level landscaped areas surrounding Building 825. The width of the open area surrounding the building varies from 6 to 27 feet wide.

Numerous utilities pass through the landscaped area between Building 825 and the above-referenced retaining wall, including an existing earthen tunnel, sewer lines, water lines and drain lines. The preliminary information provided by OAI indicates that the center line of the tunnel is about 17 feet from Building 825 and about 10 feet from the face of the retaining wall. We understand that the tunnel is about 3 feet wide by 4½ feet tall and that its invert in this area is about 13 feet deep.

A storm drain manhole provides access to the tunnel and allows 12-inch and 14-inch diameter drain lines to discharge into the tunnel. The plans provided by OAI indicate that the access manhole is about 5 feet in diameter and shows inverts of Elev. 820 and Elev. 822 for the 12-inch and 14-inch diameter drain lines. The invert of the manhole is indicated at about Elev. 815, or about 18 feet below the existing ground surface.

Little information is available with regard to the design and construction of the existing manhole, or its current condition. We understand that the manhole is a standard concrete manhole which does not include a bottom section within the unlined tunnel, and that it is assumed that it is being held in place by friction and adhesion between the side of the manhole and the surrounding soils.

The preliminary design schemes indicate that the existing manhole will be permanently covered with a concrete slab and the manhole will be abandoned in-place. The slab will be about 12 inches in thickness and will be covered with 1 to 2 feet of soil to allow landscaping over the area.

In addition to the covering of the existing manhole, the Design Storm Water Drainage Upgrades project will include an infiltrator bed, which will be constructed in an open area on the southern side of Building 825. The area is between the existing parking lot for Building 825 and the existing aircraft hardstand. The ground surface in this area is relatively level at about Elev. 832 and is an open lawn. The preliminary information provided by OAI indicates that the infiltration beds will cover an area of about 60 feet by 100 feet in plan dimensions.

Subsurface Investigation and Percolation Testing – One test boring was drilled to a depth of 12 feet below the existing ground surface in the area of the infiltration beds, while another boring was drilled to a depth of 18 feet adjacent to the existing drainage manhole behind Building 825. In addition, a percolation hole was drilled to a depth of 9 feet near the boring in the area of the infiltration beds. Their approximate locations are shown on the Site and Boring Location Plan, Figure 2, in the attached Appendix.

The borings and percolation hole were drilled on August 27, 2009 with a SIMCO 2400 truck-mounted drilling rig advancing 4-inch diameter continuous flight augers. Relatively undisturbed samples of the subsurface soils were obtained at selected depths with a 3.0-inch O.D. split-spoon sampler driven by a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler the final 12 inches into the soil is shown on the Boring Logs, Figures 3 and 4 in the Appendix, together with the materials encountered. The log of the Percolation Hole is shown as Figure 5. A Boring Log Legend is included as Figure 6 for reference.

The percolation hole was filled with water and saturated overnight. A percolation test was performed at the bottom of the hole on August 28, 2009. The test was performed in general conformance with the Site Evaluation criteria of the State of Hawaii, Department of Health (DOH) under Chapter 11-62 of the Hawaii Administrative Rules (HAR). The results of the percolation test are shown on Figure 7. A percolation rate of 32 minutes per inch (mpi) was obtained.

Laboratory Testing – Selected samples of the subsurface soils obtained from the borings were tested to determine their pertinent general engineering characteristics, including in-situ moisture content and density. Atterberg Limits tests were performed on visually representative samples to aid in the classification of the soils. The results of the laboratory tests are shown on the Boring Logs. The Atterberg Limits test results are graphically presented in Figure 8.

General Subsurface Conditions – The test borings have revealed that both the area of the infiltration beds and the existing drainage manhole are underlain by less than 8 inches of topsoil over relatively competent residual soils (soils weathered in-place from parent rock), which extend to the bottom of both of the borings at depths of 12 to 18 feet below the existing ground surface.

Groundwater or subsurface seepage was not observed in any of the borings or percolation holes during our field exploration.

The topsoil at the ground surface consists of a brown clayey silt which appears to be similar to the underlying residual soils, except that it is dry and loose. The residual soils below the topsoil consist of a highly plastic elastic silt which is classified as MH under the Unified Soil Classification (USC) system. It exhibits hard to very hard consistencies, and in general, appears to be dry to damp in the area of the infiltration beds and damp to moist in the area of the existing drain manhole. The moisture contents of the soils appear to increase with increasing depth.

Discussion, Conclusions & Recommendations – Except for the loose surface layer of topsoil, the test boring in the vicinity of the drainage manhole indicates that this area is generally underlain by hard to very hard residual soils which should provide adequate support for the proposed concrete slab over the existing manhole. The most significant geotechnical concern with the planned slab construction over the manhole is its potential effect on the existing manhole and the unlined tunnel below.

Little is known about the design, construction and current condition of the manhole, and hence there are risks associated with the new construction. These risks can be reduced but not entirely eliminated due to the number of unknowns associated with the existing manhole and its condition. Hence, we believe that it would be prudent to design the slab such that it results in minimal disturbance, and changes to the loading conditions, of the existing manhole.

The walls of the existing manhole act as unyielding retaining walls supporting the soils outside the manhole. Construction of the slab over the manhole will result in some net additional vertical load to the retained soils due to the weight of the soil and concrete over the open area of the manhole being spanned. The net vertical load would be supported by the soils surrounding the manhole which in turn would result in additional lateral pressures to the manhole walls.

The net vertical load on the surrounding soils is essentially based on the annular area of the supportive soils around the manhole; the larger the annular area supporting the slab around the manhole, the lower the net contact pressure of the slab on the supporting soils. Our analysis indicates that a 12-inch thick concrete slab extending at least 3 feet beyond the perimeter of the existing 5-foot diameter manhole, and covered with no more than 2 feet of topsoil, should result in a net additional vertical contact pressure to about 100 pounds per square foot (psf).

This net contact pressure is the equivalent of the weight of less than a foot of areal fill over the manhole, which we believe should have a minimal effect on the existing structures. The above analysis is based on using typical concrete with a unit weight of 150 pounds per cubic foot. The use of a lightweight concrete with a lower unit weight should further reduce the above estimated net contact pressure.

It is anticipated that the slab will likely be a precast slab set into place during the construction once the area at the top of the manhole has been prepared, so that forms are not needed beneath the slab and over the manhole. The supportive soil area of the slab should be cut "clean" and all loose materials removed so that compaction of the subgrade soils is unnecessary. Compaction of the soils within the annular space surrounding the manhole is not recommended as the vibrations may adversely affect the stability of the existing manhole and tunnel.

Since the existing manhole is being supported by friction between its walls and the surrounding earth, care must taken in the demolition of the upper portions of the manhole to construct the new slab and to prevent the new slab from transmitting direct vertical loads to the manhole. We believe that saw-cutting the sides of the manhole will result in the least disturbance to the existing manhole and reduce the potential of it moving downward and into the tunnel below. Alternatively, the manhole could be underpinned with small-diameter piers to prevent its movement during construction, although this would likely add significant costs to the construction.

The top of the portion of the manhole to remain in place beneath the new slab should be cut between 2 and 4 inches below the bottom of the new slab to provide a sufficient clear space and minimize the potential for the bottom of the slab contacting the top of the remaining manhole walls. The exposed soils remaining at the perimeter edge of the manhole should be carefully trimmed and cleaned off to minimize future raveling of the exposed soils.

Backfill over the slab may consist of the on-site silts or similar imported cohesive materials, provided it is free of any organics, debris, rocks or soil clods greater than 3 inches in maximum dimension, and other deleterious materials. The backfill should be placed in lifts of no more than 6 inches in loose thickness, moisture-conditioned to within 3 percent of its approximate optimum moisture content, and uniformly compacted to at least 85 percent relative compaction as determined by Laboratory Compaction Test ASTM D1557. Compaction of the backfill should be accomplished with a light static compactor to reduce the potential for vibrations into the subsurface. The use of a vibratory compactor should not be allowed.

The demolition of the existing manhole and the installation of the slab cover should be observed by Fewell Geotechnical Engineering, Ltd. (FGE) to determine whether the anticipated conditions are encountered during the construction. Intermittent density tests should be taken on the backfill to determine whether the specified levels of compaction are consistently attained. A sample of the proposed backfill material should be submitted to FGE at least 7 working days prior to its intended job-site use to allow sufficient time for testing, evaluation and approval. The recommendations presented in this report are contingent on adequate observation and testing of the geotechnical phases of the construction by FGE.

Limitations - This report has been prepared for the exclusive use of **Okahara & Associates, Inc.** for the **Building 825 Storm Water Drainage Upgrade** at Wheeler Air Force Base in Wahiawa, Oahu, Hawaii. In the performance of the investigation and the preparation of this report, we have strived to perform our services in a manner consistent with that level of care and skill ordinarily exercised by members of the geotechnical profession practicing under similar conditions in Hawaii. No other warranty, either expressed or implied, is made.

The analysis, conclusions, and recommendations submitted in this report are based in part upon the data obtained in the test borings and upon the assumption that the subsurface conditions do not deviate from those observed. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the present time, FGE should be notified so that supplemental recommendations can be given. The recommendations and conclusions contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report modified or verified in writing.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by soil samples, test borings, or test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Some contingency funds are recommended to accommodate such potential extra costs.

The site investigation may not have disclosed the presence of underground structures, such as cesspools, drywells, storage tanks, etc. that may be present at the site. Should these items be encountered during construction, FGE should be notified to provide recommendations for their disposition.

The scope of work for this investigation was limited to conventional geotechnical services and did not include archeological or environmental assessments or evaluations. Silence in the report regarding any archeological or environmental aspects of the site does not indicate the absence of potential archeological or environmental problems.

The boring locations and percolation hole were staked out in the field by FGE based on visual references at the site and the preliminary plans provided by Okahara & Associates, Inc. Ground surface elevations were estimated from the undated Topographic Plan provided by OAI on September 16, 2009. The locations and elevations of the borings and percolation hole should be considered accurate only to the degree implied by the methods used.

Groundwater was not found in any of the test borings and percolation hole of this investigation. However, it must be noted that fluctuations in the level of the groundwater, or subsurface seepage, may occur due to variations in rainfall and other factors not present at the time the measurements were made.

FGE should be provided the opportunity for general review of the final design drawings and specifications to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design and specification. If FGE is not accorded the privilege of making this recommended review, it can assume no responsibility for misinterpretations of the recommendations.

FGE should also be retained to provide soil engineering services during construction. This is to observe compliance of the design concepts, specifications, and recommendations and to allow design changes in the event the subsurface conditions differ from that anticipated prior to construction. The recommendations contained herein are contingent upon adequate construction monitoring of the geotechnical phases of the construction by FGE.

We appreciate the opportunity to be of service and look forward to continuing to work with you on this project. Should you have any questions regarding this report, or if we can be of additional service, please do not hesitate to contact us.

Respectfully submitted,

FEWELL GEOTECHNICAL ENGINEERING, LTD.

By Alan J. Shimamoto, P.E.

/ajs:tjc:fse

Attachment: Appendix

ALAN J. SHIMAMOTO
LICENSED PROFESSIONAL ENGINEER
No. 4393-C
HAWAII, U.S.A.

APPENDIX

Subsurface Investigation Summary

Project Designation: Building 825 Storm Water Drainage Upgrade **File:** 2945.01
Wheeler Air Force Base

Location: Wahiawa, Oahu, Hawaii

Project Location Map: Figure 1

Boring Location Plan: Figure 2

Drilling Equipment: SIMCO 2400

Boring Summary:

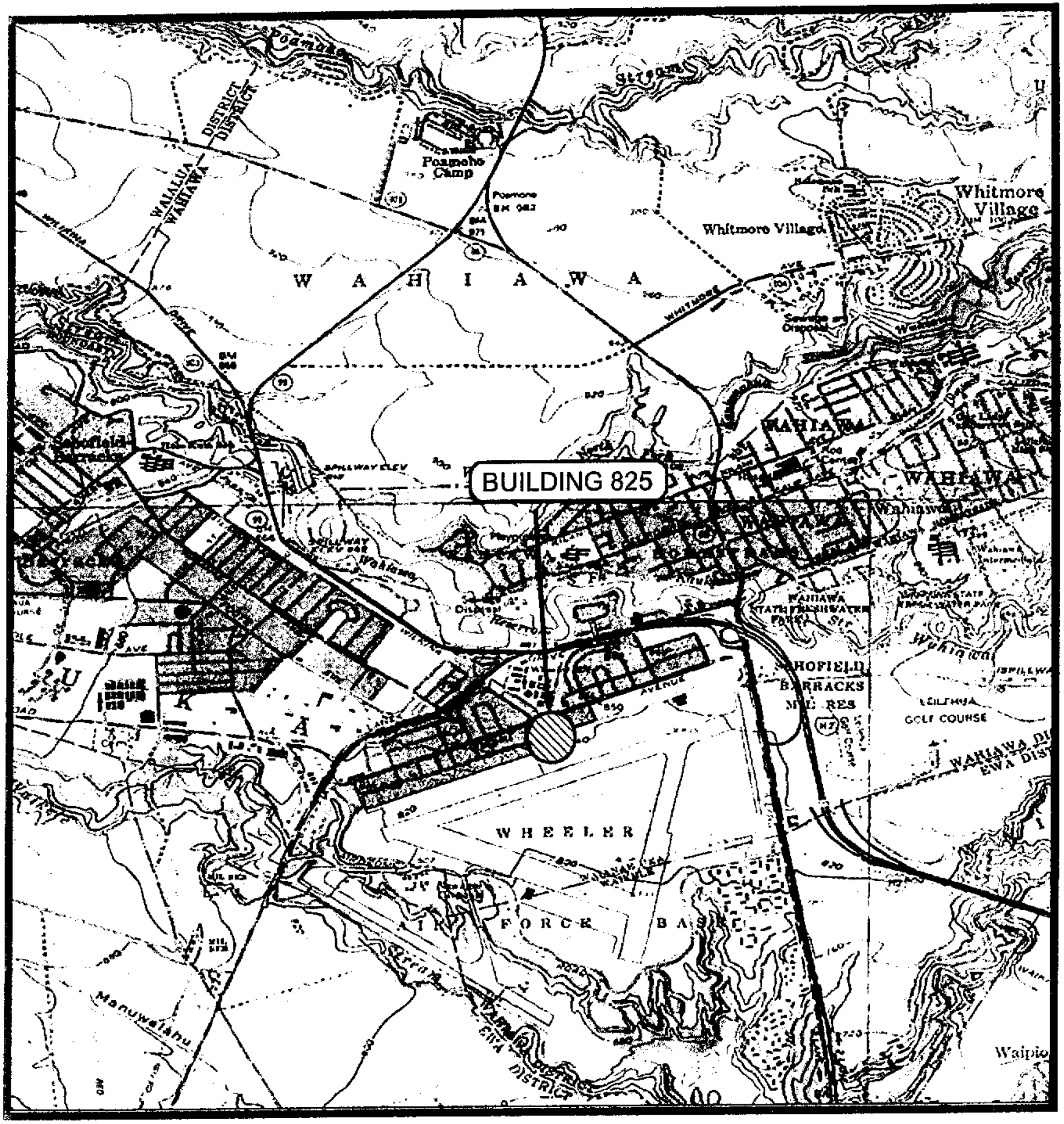
Boring/Perc. Design.	Depth (feet)	Number of Samples	NX Core	Depth to Water Table*	Figure Design.
1	12.0'	3		N.E.	3
2	18.0'	6		N.E.	4
P1	9.0'	0		N.E.	5
Totals:	39.0'	9			

Date Started: 8-27-09 **Date Completed:** 8-28-09 * N.E. = Not Encountered

Boring Log Legend: 6

Percolation Test Results: 7

Laboratory Testing Summary:	Sample No.	Figure Design.
Plasticity Chart:	1-3	8
	2-2	8



LEGEND:

PROJECT LOCATION

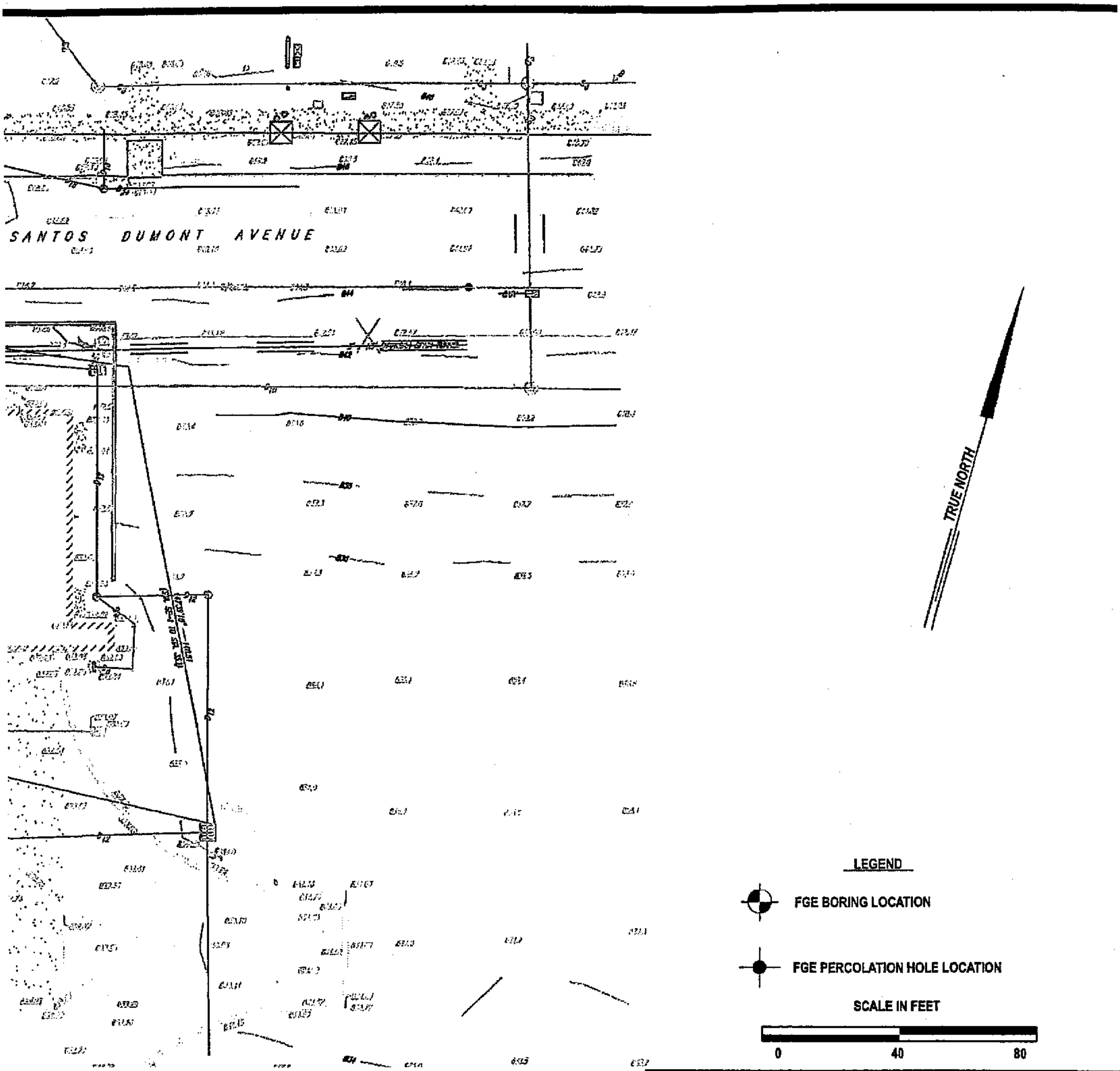
SCALE: 1:24000

GENERAL AREA:
SCHOFIELD, OAHU, HAWAII

REFERENCE:
WAIPAHU QUADRANGLE
U.S.G.S. TOPOGRAPHIC MAP

F.G.E. Ltd.	PROJECT LOCATION MAP Building 825 Storm Water Drainage Upgrade Wheeler Air Force Base Wahiawa, Oahu, Hawaii	File: 2945.01 September 2009

Figure 1



F.G.E. Ltd.

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PEARL CITY
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FEWELL GEOTECHNICAL ENGINEERING, LTD.

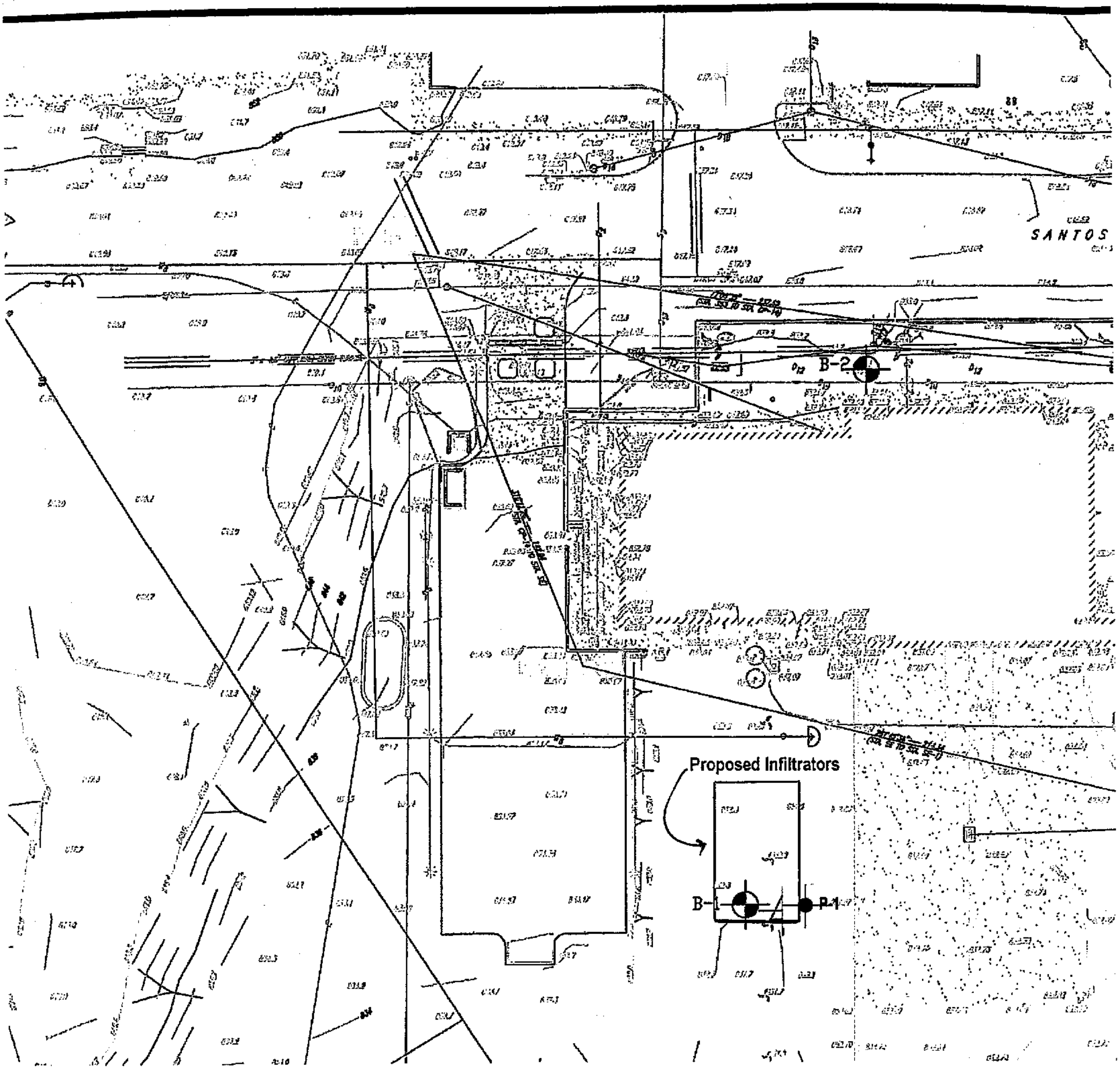
SITE AND BORING LOCATION PLAN

BUILDING 825 WATER DRAINAGE UPGRADE
WHEELER AIR FORCE BASE
WAHIAWA, OAHU, HAWAII

FILE: 2945.01

SEPTEMBER 2009

FIGURE 2



REFERENCE: UNDATED TOPOGRAPHIC PLAN PROVIDED BY OKAHARA ASSOCIATES VIA EMAIL ON SEPTEMBER 16, 2009

F.G.E. Ltd.
96-1416 Waihona Place
Pearl City, Hawaii

Boring: 1		**File:** 2945.01	
Project: Building 825 Storm Water Drainage Upgrade			
Location: Wheeler AFB, Oahu, Hawaii		**Project Engineer:**	AS
Surface Elevation:	832' ±	**Field Engineer:**	TC
Depth to Water:	None Encountered	**Drafted by:**	KL
Date Completed:	8-27-09	**Date of Drawing:**	September 2009

LAB TEST RESULTS	MOIST CONT. %	DRY DEN. PCF	BLOWS PER FT.	SAMPLE	DEPTH	CLASSIFICATION
					0	Brown Clayey Silt (MH), hard, dry
			40	1		
					5	
			25	2		Grades to Reddish Brown, dry to moist
					10	
LL=67, PI=24			31	3		(RESIDUAL)
					12	(BOH @ 12.0')
					15	
					20	
					25	
					30	
					35	

Figure 3

F.G.E. Ltd.
96-1416 Waihona Place
Pearl City, Hawaii

Boring:	2	**File:**	2945.01
Project:	Building 825 Storm Water Drainage Upgrade		
Location:	Wheeler AFB, Oahu, Hawaii	**Project Engineer:**	AS
Surface Elevation:	833' ±	**Field Engineer:**	TC
Depth to Water:	None Encountered	**Drafted by:**	KL
Date Completed:	8-27-09	**Date of Drawing:**	September 2009

LAB TEST RESULTS	MOIST CONT. %	DRY DEN. PCF	BLOWS PER FT.	SAMPLE	DEPTH	CLASSIFICATION
						Brown Clayey Silt (MH), loose, dry **(FILL/TOPSOIL)**
	31	85	27	1		Brown and Gray Clayey Silt (MH), very hard, dry to damp
LL=52, PI=38	27	83	35	2		
					5	Grades damp to moist
	43	78	27	3		
	44	76	41	4		
					10	
			47	5		
					15	
			67	6		**(RESIDUAL)**
						(BOH @ 18.0')
					20	
					25	
					30	
					35	

Figure 4

F.G.E. Ltd.

Perc. Hole: P-1
File: 2945.01
Project: Building 825 Storm Water Drainage Upgrade -
Location: Wheeler AFB, Oahu, Hawaii
Project Engineer: AS
Surface Elevation: 832' ±
Field Engineer: TC
Depth to Water: None Encountered
Drafted by: KL
Date Completed: 8-27-09
Date of Drawing: September 2009

LAB TEST RESULTS	MOIST CONT. %	DRY DEN. PCF	BLOWS PER FT.	SAMPLE	DEPTH	CLASSIFICATION
					0	Brown Clayey Silt (MH), hard, dry
					5	Grades to Reddish Brown, dry to moist
						(RESIDUAL)
					9.0	**(BOH @ 9.0')**
					10	
					15	
					20	
					25	
					30	
					35	

Figure 5

MAJOR ROCK TYPES

BASALT

TUFF

DECOMPOSED ROCK

CORAL

SECONDARY CLASSIFICATION

GRAVELLY

SANDY

SILTY

CLAYEY

MAJOR SOIL TYPES

GRAVEL

SAND

SILT

CLAY

PEAT/ORGANICS

SAMPLING SYMBOLS

3" O.D. Undisturbed Sample

3" O.D. Disturbed Sample

2" O.D. Standard Penetration Sample

No Recovery

Shelby Tube

Bag Sample

Core

Water Level

F.G.E. Ltd.	**BORING LOG LEGEND** Building 825 Storm Water Drainage Upgrade Wheeler Air Force Base Wahiawa, Oahu, Hawaii	File: 2945.01 September 2009

Figure 6

Site Evaluation/Percolation Test

Percolation Test: P-1
Date/Time: August 28, 2009, 9:00 am to 12:00 pm
Test Performed By: Fewell Geotechnical Engineering, Ltd.
Project: Building 825 Stormwater Drainage Upgrade
Owner: Hawaii Air National Guard
Tax Map Key: ____

Elevation: 832± feet
Depth to Groundwater Table: None Encountered
Depth to Bedrock (if observed): None Encountered
Diameter of Hole: 4 inches
Depth to Bottom of Hole: 9 feet below grade
Time Length Presoaking Test Hole: 24 hours

Depth Below Grade	Subsurface Profile
0 to 9 feet	Brown Clayey Silt (MH), hard, damp to moist (Residual)

Percolation Readings

Time 12 inches of water to seep away: >60 minutes (first trial – pre-soak on 8/27/09))
Time 12 inches of water to seep away: >60 minutes (second trial)

Final 2 hours of readings:

Time Interval (minutes)	Drop in Inches	Time Interval (minutes)	Drop in Inches
10	0.31	10	0.31
10	0.25	10	0.31
10	0.31	10	0.38
10	0.31	10	0.31
10	0.25	10	0.31
10	0.31	10	0.31

Average Percolation Rate (time/final water level drop): 32 min/inch

As the engineer responsible for gathering and providing site information and percolation test results, I attest to the fact that the above site information is accurate and that the site evaluation was conducted in accordance with the provisions of Chapter 11-62, "Wastewater Systems".

	Sample ID	Depth (ft)	LL	PL	PI	Classification
●	1 - 3	10.5	67	43	24	Brown Clayey Silt (MH)
■	2 - 2	3.0	52	38	14	Light Brown Clayey Silt (MH)

F.G.E. Ltd.

PLASTICITY INDEX CHART

Building 825 Storm Water Drainage Upgrade

Wheeler AFB, Oahu, Hawaii

File: 2945.01

September 2009

Figure 8